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A case history study on causation of the landslide in Santa Clara, California, USA

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ABSTRACT

This paper presents a case history study on the geologic investigation and numerical modeling of a reactivated landslide in the County of Santa Clara, California to identify the failure mechanism. The landslide occurred on an approximately 18.3-m high, north-facing slope during March 2011. The landslide measured about 33.5 m in width and about 51.8 m in length. Along the toe of the slope, a residential structure with a swimming pool was built on a cut and fill pad and there are several other structures present along the western side of the pad. The landslide occurred immediately to the south of the residential building and moved northward between the County Road A and the house's side yard. The movement of the landslide resulted in damaging the west-bound traffic lane of County Road A and encroached onto the paved driveway for the residential property. An investigation was performed to identify the failure mechanism of the landslide to conclude whether Road A re-alignment by the County or prominent cutting performed along the lower portion of the slope by the homeowner during 2000 through 2004 contributed to the reactivation of the old landslide deposit. The investigation included site reconnaissance, reviewing available published geologic information, reviewing site-specific geologic and geotechnical data developed by other consultants, and performing numerical modeling. The outcomes of the investigation indicate that the primary causation for the reactivation and failure of the subject pre-existing landslide is the prominent cutting performed along the lower portion of the slope during 2000 through 2004 and water tank cut bench. The Road A re-alignment did not contribute to the reactivation of the old landslide deposit.

1. Introduction

The landslide is located within the County of Santa Clara in the San Francisco Bay Area, California, as shown in Fig. 1. The landslide occurred on an approximately 18.3-m high, north-facing slope that separates the east/west trending Country Road A to the south from the residential structure. Along the toe of the slope, the residential structure is built on a cut and fill pad and there are several other structures present along the western side of the pad including a detached storage shed, a pool house, several large planter boxes, a barbecue area, a swimming pool, a spa, concrete-filled solar array panels, and a concrete patio. A relatively steep and paved driveway provides access from Road A to the residential structure.

The landslide occurred immediately to the south of the residential structure and moved northward between Road A and the house's side yard. The landslide measured about 33.5 m in width and about 51.8 m in length. The slope area to the east of the landslide and upslope of the access driveway appeared to have been cut. The lowermost portion abutting the driveway has been cut and over-steepened. The cut slope portion at the northeast corner of the landslide and the driveway exceeds 9.1 m in vertical height with a steep gradient of 1 horizontal to 1 vertical (1H:1V) as shown in Fig. 2. A relatively narrow mid-slope cut bench with an associated 0.9- to 1.5-m high cut slope above the bench extends westward at near mid-height between the driveway and Road A until it merges with an approximately 6.1-m wide level bench cut across the landslide where the tank was situated. A circular water tank occupied the western part of the noted bench. This bench and the water tank were displaced downslope as the landslide moved northward. An approximately 3- to 3.7-m high cut slope, with slope gradients varying between about 0.5H:1V and 1H:1V, that was generated as a result of the water tank cut bench borders the bench along its upper (south) side.

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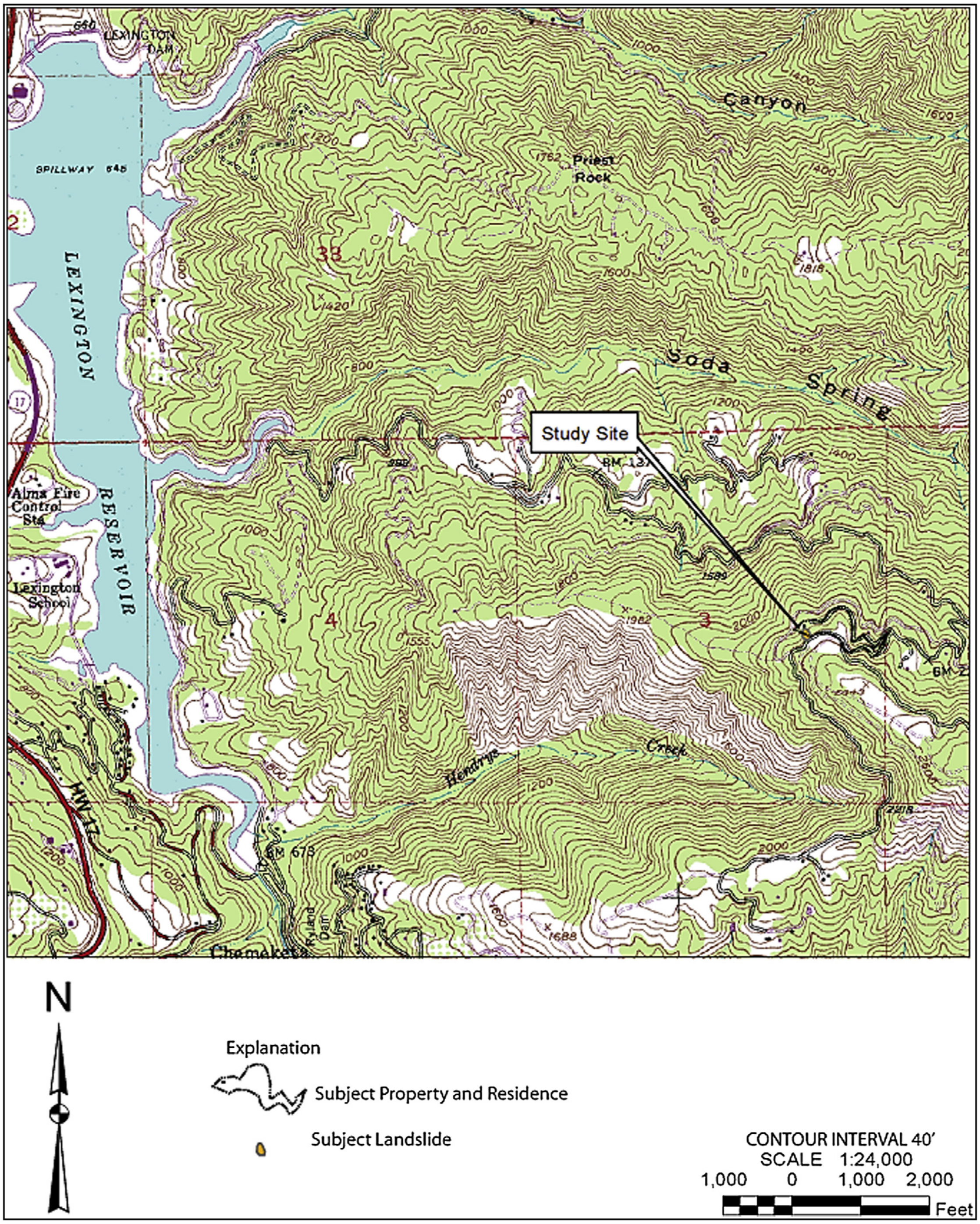

Fig. 1. Location of the 2011 landslide (After McLaughlin et al., 2001).

The topographic contours within the limits of the landslide as shown in Fig. 2 depict the topography of the body of the landslide in 2012 after the geomorphic expression of the slope failure had been altered by toe cuts, pioneered access routes for the subsurface exploration equipment in addition to activities related to the soil nail wall construction along the landslide's headscarp. The surveyed topographic contours from the July 2011 topographic survey conducted by Westfall were superimposed to the area of the water tank and its cut bench. The survey was performed after the failure but before the subsurface exploration took place to show the extent and magnitude of cut associated with the water tank construction. Although this noted level bench and associated cut slope had been displaced northward as the landslide moved downslope, their relative measurements and gradients remained largely intact.

Beyond the western landslide margin of the landslide to the west, the toe of the slope abutting the created level area where the sheds and planters are situated has been cut steeply to a near-vertical gradient that varied in vertical height from about 0.9 m to 2.4 m. A concrete stairway that was cut ascending the slope is present abutting the northwest corner of the landslide at the western landslide margin. Near mid-height of the slope to the west of the landslide, another cut bench that measured up to about 6.1 m

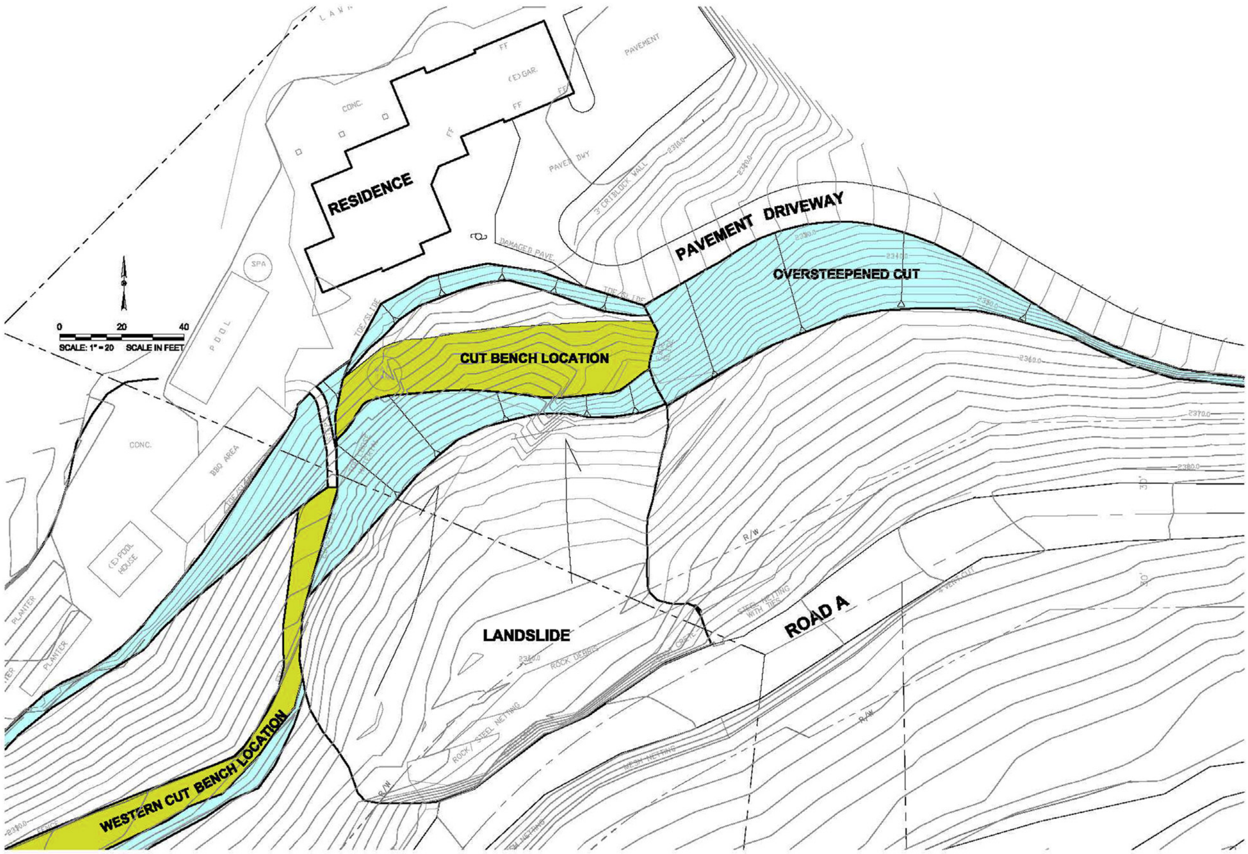

Fig. 2. Site plan.

in width extends westward across the slope between the concrete stairway and the western end of the created level area where the shed and the noted planters are located. The extension of the western cut bench has resulted in an associated slope that is up to 3 m in vertical height and a relatively steep 0.5H:1V to 1H:1V gradient.

2. Geologic setting and site-specific geology

The area of the landslide is dominated by ridge lines that trend east/west to northwest with steep side slopes and intervening valleys. The ground elevation along the top of the landslide at Road A is about 723.3 m above mean sea level (MSL) and at the toe of the landslide it is about 704 m MSL. The area of the landslide and residential pad is situated near the top of a spur ridge with side slopes that dip steeply towards the west and north. The area is heavily vegetated with trees and dense brush.

The geology of the site area has been mapped by several mappers that include Dibblee and Brabb (1978), McLaughlin et al. (1991, 2001), and the CGS (2002a, 2002b). They generally agreed that the area is underlain by the mélange unit of the Franciscan Complex (KJfm). The mélange unit is generally composed of a highly sheared sandy to clayey ground up shale matrix that supports an undifferentiated collection of varying types and sizes of rock fragments. The rock types in this area include metasediments (shale and sandstone), metavolcanics, chert, and limestone among others that range in size from several feet to blocks and slabs that extend for more than a mile.

The Cretaceous/Jurassic KJfm unit is generally not differentiated, but in the site area, it seems to be primarily composed of metamorphosed sandstone and shale. Dibblee and Brabb (1978) and McLaughlin et al. (1991) showed the bedrock bedding to strike east/west to northwest and dip adversely about 30° and 60° to the north and northeast, respectively. It was observed and measured that adverse bedrock bedding along the near-vertical cuts behind the shed area to the west of the landslide trends about 15° east of north and dips about 21° downslope to the northwest.

McLaughlin et al. (1991, 2001) mapped several large landslide deposits along the north-facing slopes to the north and east of the site area in the Franciscan mélange terrain, which is considered to be highly susceptible to landslide activities and slopes underlain by Franciscan mélange have a high potential for failure. The CGS (2002b) showed the entire area of the adjacent ravine (swale) to the west and the upper portion of the subject landslide area (immediately downslope of Road A) to be within the limits of zones of required investigation associated with earthquake-induced landslides where "Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693(c) is required".

The Seismic Hazard Zonation Report 069 for the Los Gatos Quadrangle, prepared by the CGS (2002a), presents a landslide inventory map of the area and it shows large-scale landslide deposits immediately to the east, north and west of the site. The above-noted swale to the west of the site is shown to be entirely underlain by a large landslide complex. The site area is shown to be along the head margin of the noted large landslide complex. The CGS's landslide inventory map is presented herein as Fig. 3.

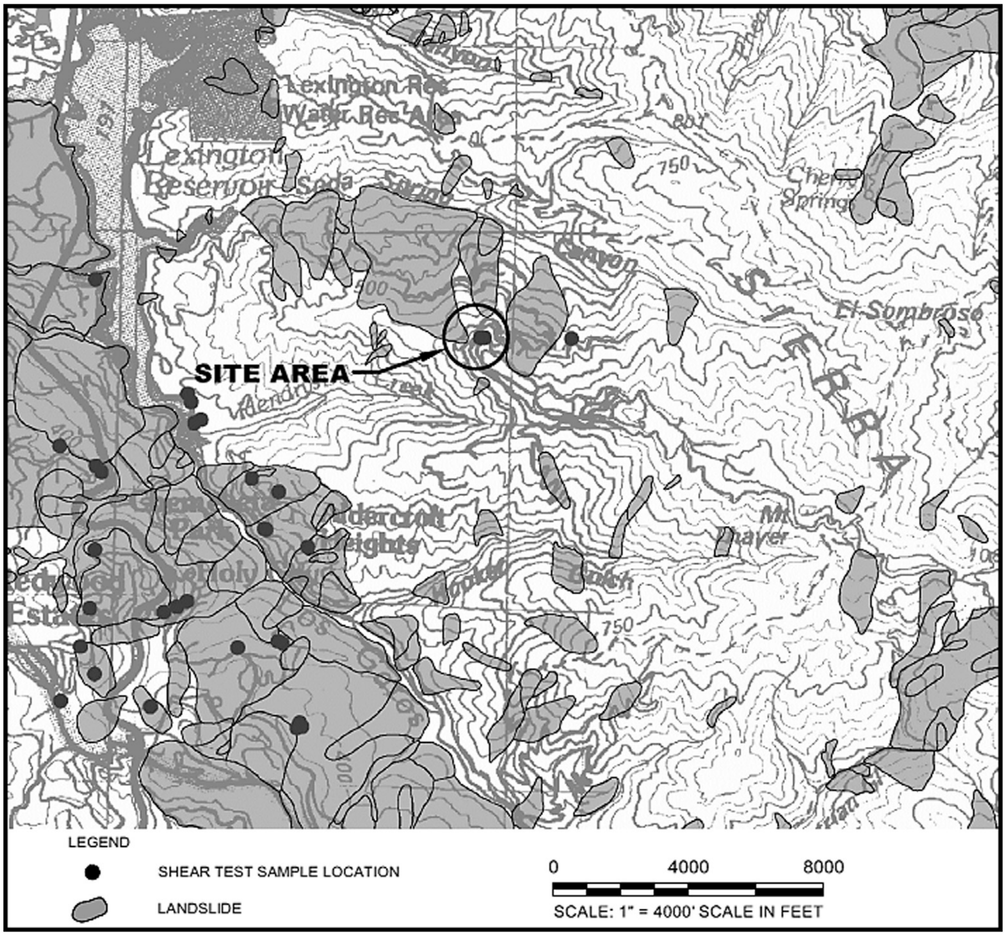

Fig. 3. Landslide inventory map (After CGS, 2002a).

Furthermore, the entire site and surrounding area are situated within the County of Santa Clara geologic hazard zones associated with landslides (County of Santa Clara, 2012).

3. Review and interpretation of aerial photograph

In order to have a good understanding of the history of the site and historical activities at the site, abundant historical aerial photographs were reviewed. Detailed descriptions of review and interpretation of aerial photographs can refer to the report by Kleinfelder (2014). A brief discussion on the review and interpretation of selected aerial photographs is summarized in the following sections.

The oldest aerial photographs are available from 1940s. The 1940s and 1950s photographs show an irregular-shaped lot with a structure on it. The roadway and pad area are surrounded with dense brush and heavy vegetation growth. The 1960s photos show that the lower portion of the slope separating the pad from Road A has been cleared of vegetation. The 1980 photos show that the western cut bench (Fig. 2) has been cut across the slope and extended westward. A cut along the head of the topographic swale is also extended westward to connect with the western end of the western cut bench. The 1985, 1987 photos show a second driveway cut across the slope higher than the one noted above on the 1980 photos. The cut extends across the slope towards the prominent U-turn where Road B and Road A meet.

The 1990 photos show a well-defined, linear cut along the toe of the slope where the failure occurred and along the top of the prominent topographic swale. The 1999 photos show fresh clearing of the pad area and the removal of the structure. Minor encroachment on the toe area is visible.

The 2000 stereoscopic photos show a driveway graded across the slope roughly where the higher 1985 driveway was and it makes a prominent U-turn behind (south of) the newly built house. A significant cut bench was extended which we label on Plate 2 as the water tank cut bench. The water tank has not been placed on the bench yet. Asphalt patching is clearly visible on the 2000 photos along the in-board side of Road A immediately above where the significant cut was made downslope for the driveway. The asphalt patch extended into the area and bent on Road A where the landslide subsequently occurred. A second smaller asphalt patch was observed along Road A to the east upslope where the driveway cut was made. This is interpreted as the slope's initial response in the form of distress to the prominent cut and lateral support removal made downslope of the roadway.

The May 2002 photo shows the water tank cut bench to have been extended westward and turned slightly southwestward to conform to the slope and it also shows the water tank in place. Road A seems darker than that in 1999 and the earlier discussed asphalt patches are not visible anymore indicating possible re-surfacing or re-paving.

The post May 2002 stereoscopic photos show the water tank remained in place and the fill along the top of the topographic swale to the west enlarged and its slope rounded off. A prominent cut slope is visible below the tank and the toe of the cut slope is also visible behind the house extending from near the tank to the driveway and curving to accommodate the new and significant driveway cut, which had been made to shift the driveway northward and lower its elevation.

The October 2004 photo shows the toe of the slope has been cut further and its position shifted southward closer to the water tank than it was in the August 2003 photo. The May 2006 photo shows asphalt patching at the bend generally coinciding with the headscarp of the 2011 landslide.

The June 2011 photo shows what appears to be a white pipe extending up the diagonal cut bench/trail noted above. The soil nail wall and cut slope along the south side of the roadway are also visible along with the water tank displaced downslope. The toe of the slope has shifted and partially damaged the upper edge of the paved driveway pavement and water staining/siltation seems to extend from the toe of the slope all the way down the driveway to near its eastern end.

4. Past geotechnical investigation

Before we were retained to perform an investigation to explore the causation of the 2011 landslide, a geotechnical investigation was performed by Consultant A in 2011 right after the landslide occurred. One of the purposes of this study was to comment on the conclusions made in the geotechnical investigation report by Consultant A. In the following sections, a brief description and comment on their investigation and conclusions is summarized.

4.1. Field investigation

In 2011, Consultant A performed a subsurface exploration program consisting of three exploratory borings near the lower and upper portions of the landslide, respectively. The locations of these borings are shown in Fig. 4. The subsurface exploration encountered pervasively sheared Franciscan Complex rocks consisting of sandstone and basalt in borehole LDB-1 and predominantly sandstone in LDB-2. Free groundwater was encountered in B-2 and LDB-1 at about 3.7–4 m below the ground surface. Based on their field investigation, a cross-section with interpreted subsurface conditions was developed by Consultant A for their numerical modeling as shown in Fig. 5.

The Franciscan mélange generally consisted of highly weathered, pervasively sheared and highly fractured earth material. In the immediate area of the subject landslide it is comprised of adversely dipping sandstone. The mélange has a high potential for slope failures and is notorious for sliding because of its highly sheared nature and lack of structural continuity which make it characteristically unstable.

During the subsurface exploration, a 0.15-m thick layer of saturated highly plastic clayey gauge was observed in both the upper and lower portions of the landslide. It is their opinion that the relatively thick clayey gauge layer formed the base of the subject landslide and resulted from multiple episodes of deformation in the past.

4.2. Causation of the 2011 landslide

Based on the results of their field investigation and numerical modeling, Consultant A concluded that the clayey gauge layer is a relict of an earlier episode of deformation which is either landslide or fault related which we do not agree with. They believed that the causation of the 2011 landslide was due to surface water infiltrating through the surface cracks along Road A. They also concluded that substantial amounts of runoff and rainfall infiltration into cracks caused the failure of the old landslide deposit after the realignment of Road A by the County of Santa Clara in 2012. In reviewing their model and slope stability analyses, we cannot concur the elevation of the groundwater on their model, which we believed was unreasonably high. Such a high groundwater cannot be backed up by available data and our local experience.

5. Our study on the causation of the 2011 landslide

We did not perform any subsurface exploration or laboratory testing as part of this current study. Instead, we reviewed existing data from the subsurface explorations by Consultant A and site aerial photographs and re-assessed the causation of the 2011 landslide.

We performed their numerical modeling of slope stability on the cross-section as shown in Fig. 5. It should be noted that Consultant A used the same cross-section for their slope stability evaluation. Unlike Consultant A, however, we have carried out their numerical modeling as follows:

(1) A zone of 0.15-m thick basal failure plane consisting of clayey gouge extending from the headscarp to the toe of the landslide was modeled.
(2) Two failure modes were modeled, including a rotational slip surface and a translational slip surface. The definitions of the rotational and translational slip surfaces are shown in Fig. 5 as recommended by Consultant A.
(3) The groundwater levels encountered in Consultant A borings were used in the slope stability analyses. Groundwater levels were assumed to remain unchanged.
(4) The shear strength properties of the basal failure plane are assumed with only a frictional angle. This assumption is validated by Stark et al. (2005) for residual drained strengths for clays with very high liquid limits and plasticity index within a pre-existing basal failure plane.
(5) The residual drained frictional angle of the basal failure plane was back calculated from pre-failure condition and then used to assess slope stability for other conditions.
(6) For the Franciscan Complex mélange material, shear strength properties were assumed to be similar to those presented in the CGS Seismic Hazard Zone Report 069 for the Los Gatos Quadrangle CGS (2002a). For the landslide (Qls) debris material, representative shear strength properties are chosen based on the soil type and the understanding of local geology. The assumed and back-calculated shear strength properties are shown in Table 1.

Slope stability analyses were conducted using the limit-equilibrium software program SLOPE/W (version 8.0.10.6504). The factor of safety (FOS) against slope failure was calculated using Spencer's method with a fully-specified slip surface constrained within the basal failure plane. Spencer's method is a two-dimensional limit-equilibrium method that satisfies force equilibrium of slices and overall moment equilibrium of the potential sliding mass.

Three cases corresponding to three different stages of the landslide were considered in slope stability analyses. The three stages of the landslide contained the stage before failure occurred and before the 2002 hillside grading occurred (i.e. "pre-failure and pre-cut"), the stage after the grading occurred in 2002 (i.e. "pre-failure and post-cut"), and the stage after the landslide movement occurred in 2011 (i.e. "post-failure"). The hillside grading that was modeled in the "pre-failure and pre-cut" included the cut for the water tank bench and the significant cut in 2002 for the driveway at the base of the hill although the initial higher driveway cut was made in 2000.

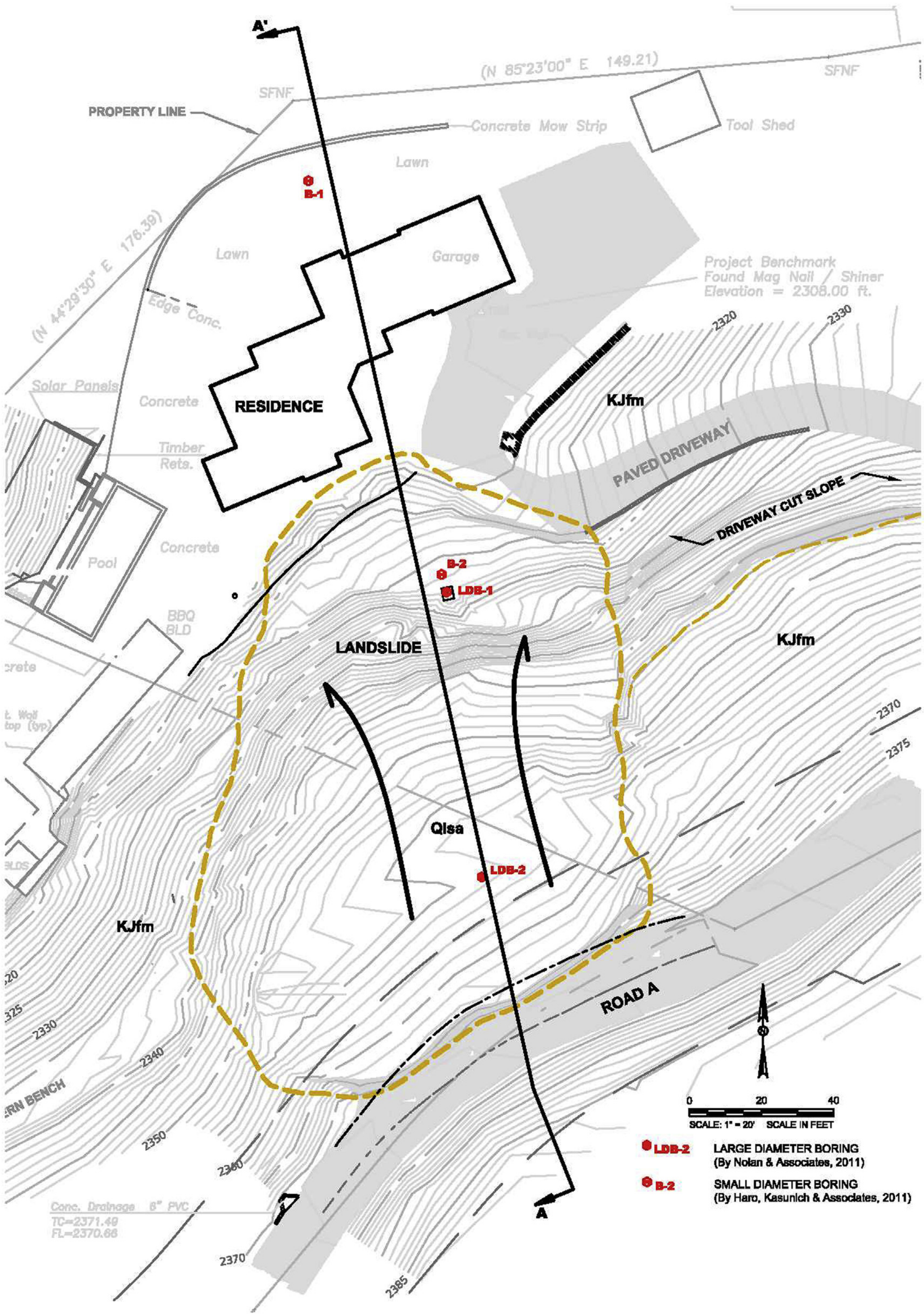

Fig. 4. Site map with boring locations.

It is worth noting that the ground surfaces that correspond to the latter two stages were derived from the cross-section shown in Fig. 5 while the first stage was generated by extending a straight line from the pre-failure toe to the top of the water tank cut slope. The post-failure ground surface topography was the only condition surveyed and the pre-failure topography was estimated by Consultant C since no survey of those conditions was available.

To determine the frictional angle of the basal failure plane, slope stability analyses were performed assuming that the FOS of pre-failure and post-cut condition topography was slightly less

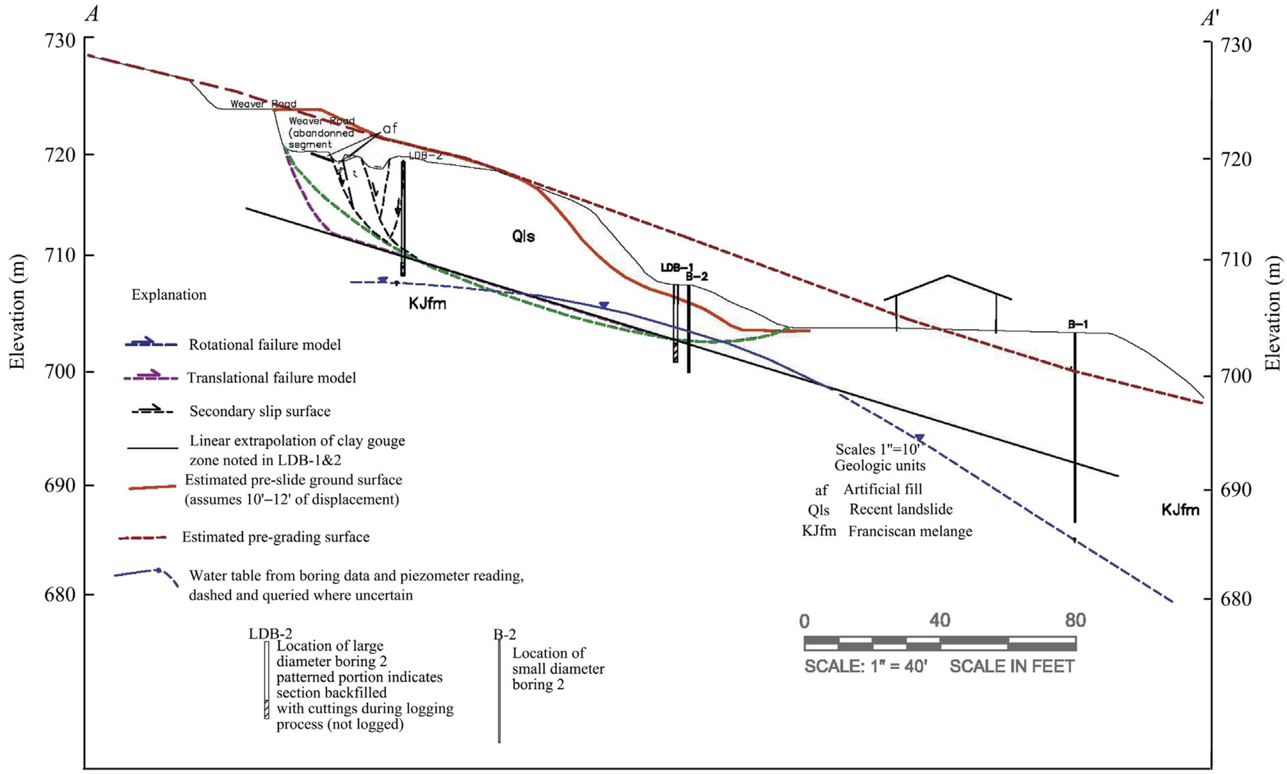

Fig. 5. Cross-section *A*–*A*′.

than 1.0. The Slope/W model developed for back-calculating the frictional angle of the basal failure plane is shown in Fig. 6. Under these conditions, a FOS of 0.99 was assumed for the pre-failure and post-cut topography to back calculate the frictional angle of the basal failure plane. Using the back-calculated frictional angle of the basal failure plane, the values of FOS for pre-failure and pre-cut and post-failure conditions were evaluated. The results of stability analyses are summarized in Table 2. The results indicate that the pre-failure and pre-cut condition was marginally stable (FOS of 1.06 or 1.04) prior to the 2002 grading. As shown in Table 2, the impact of the hillside grading reduced the FOS by 0.07 (7%, i.e. from FOS of 1.06 to 0.99) for a rotational-type failure slip surface and by 0.05 (5%, i.e. from FOS of 1.04 to 0.99) for a translational-type failure slip surface.

6. Conclusions

Based on the review of the referenced documentation, site visits, numerical modeling, and extensive experience with landslide deposits in similar formations, the primary causation for the reactivation and failure of the subject pre-existing landslide is the prominent cutting performed along the lower portion of the slope

Table 1
Summary of material properties used in the stability analysis.

Geologic material	Total unit weight (kN m^{-3})	Friction angle (°)	Cohesion (kPa)
Landslide debris (Qls)	20.4	28	16.8
Franciscan mélange (KJfm)	20.4	35	23.9
Basal failure plane	19.6	23/20.2*	0

Note: *The back-calculated friction angle for the basal failure plane is 23° and 20.2° for rotational failure mode and translational failure mode, respectively.

during 2000 through 2004 and water tank cut bench, near the concrete stairway and the toe of the slope to reshape the toe and construct the masonry block wall. The prominent cuts made along the lower portions of this pre-existing landslide have drastically decreased its resisting force and caused it to fail. On the other hand, surface infiltration into the surface cracks along Road A should not be primarily responsible for the 2011 reactivation based on our sensitivity analysis of the groundwater depth on the slope stability.

Grading to construct Road A performed early last century most likely removed the head portion of the pre-existing old landslide and decreased the driving force of the landslide. Repeated patching and sealing of the asphalt cracks by the County of Santa Clara most likely delayed its failure after the toe of the slope had repeatedly been encroached on and cut. Common knowledge in the engineering geology and geotechnical engineering profession that over-steepening and undercutting along lower portions of slopes to gradients exceeding about 2H:1V in Franciscan mélange is not considered acceptable by the Standard of Practice in this area, especially if adverse bedrock bedding occurs. The claim made by the Consultant A that water infiltration through the asphalt cracks after the re-alignment of Road A caused the failure lacks proper engineering geologic reason, judgment, and direct evidence as it is only based on speculation.

It is also concluded that the Road A re-alignment did not contribute to the reactivation of the old landslide deposit. There is clear evidence that the pre-existing landslide was showing signs of reactivation in the form of lateral displacement, vertical down-drop, distortion and buckling well before the en masse reactivation of 2011. It is important to note that in presence of significant distress reported and documented along the headscarp area at the roadway, the main sliding body of the landslide and the toe area is indicative of the old failure's triggering and reactivation and

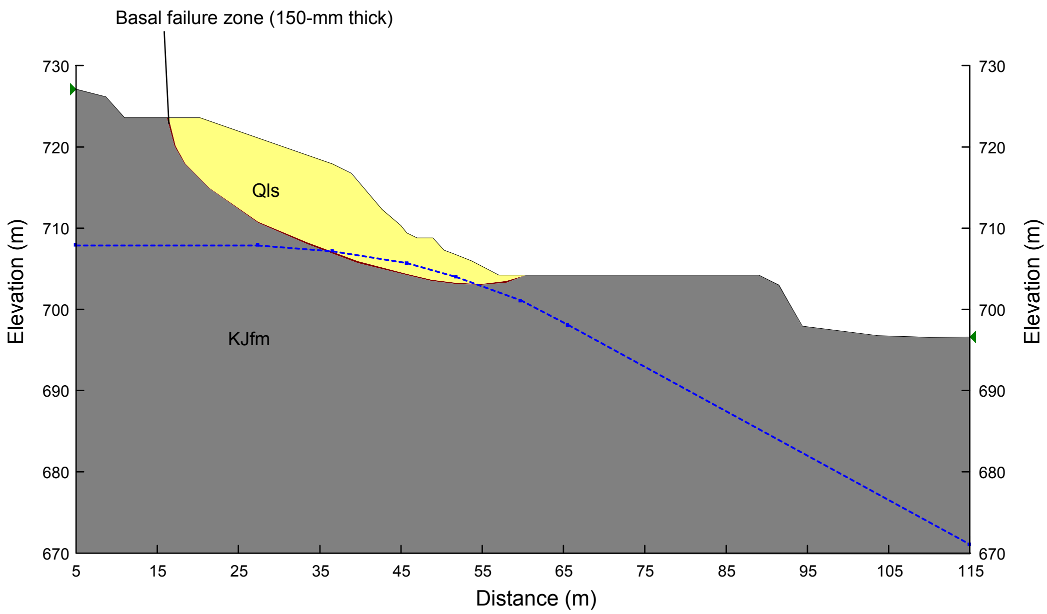

Fig. 6. Slope/W model for back calculation of frictional angle of basal failure plane.

Table 2
Summary of results of slope stability analyses (back calculation on pre-failure and post-cut model).

Case	Condition description	Failure mode	Friction angle of slip surface (°)	FOS
I	Pre-failure & pre-cut	Rotational	23	1.06
II	Pre-failure & post-cut			0.99
III	Post-failure			1.23
IV	Pre-failure & pre-cut	Translational	20.2	1.04
V	Pre-failure & post-cut			0.99
VI	Post-failure			1.19

should be considered as the beginning stages of the impending en masse failure. A slope under such conditions is considered failing and its FOS is less than 1.0 as it is already either rotating or translating down the slope along the basal failure surface.

Conflict of interest

We wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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